Structural Response of Masonry Infilled Timber Frames to Flood and Wind Driven Rain Exposure

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Abstract

In the current changing climate historic timber frame buildings are exposed to ever more severe and frequent extreme weather conditions such as floods and wind driven rainstorms. These structures are especially vulnerable to moisture ingress and subsequent decay. In light of this there is a need to better understand and quantify the impact of this exposure on the mechanical behaviour and capacity of such systems. Here an experimental investigation is presented which sets out a novel test method for measuring the impact of cyclic wind driven rain and flood exposure on the lateral stiffness and strength of masonry infilled timber frames. Empirical data presented here indicates losses in elastic stiffness exceeding 75% as a result of exposure, whilst analytical assessment confirms the failure mechanism that describes yielding of the system in weathered and unweathered states. This work has measured the extent of structural decay in direct relation to the meteorological parameters wind speed and precipitation accumulation, giving deeper, understanding of the vulnerability of the structural system of masonry infilled timber framing to these climate phenomena.

Introduction

Around the world precipitation levels are continuing to be observed to increase, linked to the rise in global mean temperature (IPCC, 2014), with ever more unprecedented accumulations leading to widespread flooding in the UK (Thompson, et al., 2017), and across Europe (Alfieri, Burek, Feyen, & Forzieri, 2015), the United States (Mallakpour & Villarini, 2015) and elsewhere across the globe (Hirabayashi, et al., 2013). Likewise there is evidence to suggest storm frequency has increased in the latter half of the 20th Century, especially in the Northern Hemisphere (Vose, et al., 2014), where observational evidence of increased storm activity in the North Atlantic since the 1970's exists (IPCC, 2007).

One consequence of this for the built environment is increased exposure of buildings to flood and storm conditions; that is strong winds, high rainfall accumulations and inundation by flood water. Historic masonry infilled timber frames (HMITF) are often highly prevalent in such affected urban centres, for example Prague (Holicky & Sykora, 2009) and York, (MacDonald, 2012). Flood and wind driven rain exposure lead to the saturation of absorbent historic façade fabric (Drdacky, 2010), such as the timber, mortar and infill material used in HMITF. Windstorms and floods expose facades to physical damage caused by loading from wind pressure (Holmes, 2015) and hydro-static or -dynamic loading (ASCE, 2006). The comparatively low strength and stiffness of HMITF leaves them potentially more susceptible to damage from wind and flood loading than modern framed buildings.

Investigation of climate change effects in relation to cultural heritage is a growing field of research application. Much work focusses on spatial risk assessments driven by observational measures of building vulnerability, such as cultural significance either through formal scheduling (Wang, 2015) or community perception (Vojinovic, et al., 2016) and other factors such as age and condition (Stephenson & D'Ayala, 2014). Technical studies which garner evidence for the physical and mechanical vulnerability of heritage buildings are less common, and offer little insight into the structural impacts of climate change on historic structures.

Impact assessments focus on physical damage such as salt induced weathering (McCabe, et al., 2013), both stone (Sass & Viles, 2010) and brick masonry (Binda, Cardani, & Zanzi, 2010) wetting and drying regimes. Whilst vulnerability indicators and scales are emerging for cultural heritage at large spatial scales (Sabbioni, Brimblecombe, & Cassar, 2012), damage functions are typically limited to physical, chemical and biological damage (EU, 2015). These are derived from visual observations rather than investigation of cause and effect relationships, and do not acount for structural impacts of climate exposure.

Work seeking to understand and quantify the loss of structural integrity due to flooding does not typically examine historic building systems. Experimental work has focussed on the study of modern masonry blockwork in either solid (Herbert, Gardner, Harbottle, & Hughes, 2012) or cavity (Escarameia, Karanxha, & Tagg, 2007) wall form. Theoretical (Kelman & Spence, 2003) or probabilistic (Mebarki, Valencia, Salagnac, & Barroca, 2012) analyses of masonry vulnerability to flood depth are not directly relatable to historic construction systems; making use of modern material paramters, geometric forms and construction details. Comparable work which addresses the behaviour of historic timber framed structures to flooding is lacking from the knowledge base, although studies investigating storm damage to traditional timber structures (Pazlar & Kramar, 2015) are beginning to emerge. More developed investigation of hazard-damage relationships for historic timber frames in earthquakes exists, including with masonry infills both with (Ferreira, Teixeira, Duta, Branco, & Goncalves, 2014) and without diagonal bracing (Duta, Sakata, Yamazaki, & Shindo, 2016).

National flood risk assessment (FRA) protocols typically compute losses on the basis of economic value. In the UK typologies relating to building purpose are used in FRA manuals (Penning Rowsell, Priest, Parker, & Tunstall, 2013), which do not account for the construction form of the exposed building stock associated with their historic nature. The equivalent US FRA model accounts for modern timber framing as a typology (FEMA, 2013), but gives no indication of its fit to historic timber frames. Damage scales embodying physical typology features often focus on building shape and form (Maiwald & Schwarz, 2012); (Kelman & Spence, 2004), with limited examples attempting to incorporate mechanical or structural parameters into the measure of vulnerability (Custer & Nishijima, 2015). With global damages from flood events leading to ever increasing costs (SwissRe, 2012), there is a pertinent need to examine the structural implications for historic structures so that future losses can be understood, predicted and appropriately managed.

The present work sets out a methodology for systematically investigating the structural response of historic masonry infilled timber frames (HMITF) to exposure to wind driven rain and flood. In the first section the experimental approach is described, which incorporates a novel methodology and test rig design for generating environmental weathering conditions for use in the laboratory as hazard scenarios. Reclaimed historic building materials and traditional construction techniques are used to produce full-scale test specimens.Following environmental exposure structural testing is carried out to determine the mechanical response of the HMITF after weathering from flood and wind driven rain. The second half of the paper presents the empirical findings and the results of a theoretical assessment of the HMITF's under weathered conditions.

The case study location of Tewkesbury in Gloucestershire, a location prone to wind driven rain exposure, and which has suffered severe flooding (Figure 1) in recent decades (Marsh & Hannaford, 2007) was studied. The work forms part of the Parnassus project (www.ucl.ac.uk/parnassus), which took an interdisciplinary approach to the investigation of risks posed to the historic building stock from climate change. The project incorporated an on-site monitoring campaign to measure hygro-thermal effects of wind driven rain and flood on building fabric through the collection of concurrent climatic and materials response data (Aktas, D'Ayala, Erkal, & Stephenson, 2015). Work to determine the probabilistic flood risk at the site highlighted the significance of high resolution modelling in ascertaining building level exposure (Smith, Bates, Freer, & Wetterhall, 2014). Meanwhile a statistical study based on observations of building typologies led to the derivation of a vulnerability model for wind driven rain and flood exposure to historic structures (Stephenson & D'Ayala, 2014). The work presented here extends the outputs to the study of structural vulnerability of HMITF to flood and wind driven rain exposure.

Experimental Investigation

From Climate Hazard Data to Experimental Test Conditions

Review of fundamental knowledge and laboratory test protocols that focus on wind driven rain and flood exposure highlighted that whilst extensive understanding of these phenomena exist, experimental studies have tended to derive from simplistic recreation of the weathering conditions. Wind driven rain (WDR) exposure has been modelled and computed for many years, a review of which is provided by Blocken and Carmeliet (2010) and Stephenson (2016). Early work by Lacy (1971) led to the derivation of the empirical relationship shown below, which describes the relationship between wind speed (U), rainfall (R_h) and wind driven rain (R_{wdr}), the constant κ accounting for the maximum speed a raindrop will fall given terminal velocity effects.

$$R_{wdr} = \kappa \, U \, R_h \qquad (1)$$

Translation of this analytical formula into a measure of WDR exposure exists in the form of exposure maps published in BS 8104 (BSI, 1992), and more recently in BS EN ISO 15927-3 (BSI, 2009)(BSI, 2009) where a building oriented approach is taken. This computes exposure at the individual building scale using a spell based approach, although gives guidance only on the minimum threshold of rainfall accumulation and wind speed for WDR wetting to occur. D'Ayala and Aktas (2016) present a critical appraisal of the analytical and codified models referred to above, on the basis of the wind driven rain data collected at the site in Tewksbury within the Parnassus project. Current literature reporting on laboratory tests which measure wind driven rain impacts on historic building fabric, provides wetting rates that are not directly translateable back to climatic conditions, rather provide guidance on total water volumes and test durations (Baker, Sanders, Galbraith, & Craig McLean, 2007a); (Sass & Viles, 2010).

The new method proposed herein takes reference from the location specific approach of exisiting British Standards, however looks to generate more specific weathering conditions which account for the extreme precipitation conditions which lead to flood events. This improves on the existing spell based approach by setting out exposure conditions using a finite temporal range. Meanwhile a cyclic approach provides realistic impacts of wind driven rain exposure over time, where wetting and drying cycles instigate fatigue in the construction system. This is especially appropriate for timber frame systems where cyclic exposure to moisture produce particularly damaging environmental conditions, due to the effects of moisture fluctuation on the breakdown of timber material and subsequent reduction in mechanical capacity (Stephenson, 2016).

The sequence design rationale identifies a total duration and volume of water for a given rainfall event, and disperses this using a rainfall intensity measure most likely to induce wetting in the construction materials. This reflects the fact that lower intensity rainfall allows for greater absorption of moisture into the fabric, as surface saturation effects are not significant (Hall & Kalimeris, 1982). It additionally links the rainfall to pluvial and groundwater flooding, which are those typically associated with more recurrent and prolonged floods. Whilst it is not possible to study the response of the materials and structural system at reduced temporal scale, the aim is to generate conditons suitable for studying long term effects.

The method of determining test flow rates is depicted in Figure 2 (where millimetres per square metre are equivalent to litres) and this was applied to the case study location of Tewkesbury using a 30-year daily precipitation data set covering 1981-2011, obtained from the Met Office's MIDAS system (MetOffice, 2012), from which an average daily precipitation total of 36 mm/day was calculated. This was then used in conjunction with 2m/s of wind speed, specified as the minimum wind speed generating "wetting conditions" in BS 15927-3 (BSI, 2009), and input into the WDR equation above to derive a total WDR amount of 10mm/day.

To encourage wetting of the wall the water was dispersed at the lowest possible intensity rate that correlated to the probability of occurrence of the rainfall amount. Studies by Holland (1960) highlighted the relationship between rainfall duration and intensity applicable to sites across the UK and this data set the threshold for use in this study (Figure 3). The duration of cycles was set so as to produce a feasible test procedure, which could be programmed to run automatically in a 24-hour period.

This ultimately determined a 3 hour wetting and drying cycle design of: 40 minutes of wetting, giving a flow rate of 0.375 L/minute, followed by a 2 hour and 20 minute period of drying of the wall. This individual cycle was then repeated a given number of times in order to produce a hazard scenario of given severity. Each of the wetting cycles corresponds to an approximately annually occurring wind driven rain event, such that 100 cycles represent 100 years of weathering. The flow rates used, being derived from observed data, do not account for any probabilistic climate change related increase in precipitation amounts. Rather these figures are intended to provide for a correlation of loss with weathering intensity based on measured precipitation rates.

Four HMITF’s were tested in total, a control specimen tested with no wetting (Frame 1) and three further specimens tested under different conditions. Frame 2 was subject to a total of 100 cycles followed by a 100 year return period flood. Frame 3 was subject to this procedure twice, allowing for drying to original moisture content between each repetition. A final specimen, Frame 4, was used to test whether more dispersed cycling would pose greater risk to the structure. This was achieved by interrupting the weathering after every 10 cycles and drying the wall to original moisture content, continuing until 100 cycles had been applied and completing the test with the same 100 year return period flood. In each case the depth of flood was 0.75m for a period of 72 hours. These conditions were extracted from the flood model derived within the Parnassus project (Smith, Bates, Freer, & Wetterhall, 2014).

A bespoke test rig (Figure 4) was constructed to facilitate the weathering whilst also allowing for the full-scale specimens to be continuously loaded vertically in compression, to represent in-situ dead loading. The rig comprised the following principal components:

1. Hanging frame capable of supporting and measuring with electronic load cells the weight of the test specimens to a resolution of 500g continuously throughout the test procedure, to monitor the weight increase attributable to moisture ingress.
2. Steel plates situated immediately above and below the specimens linked with steel bars to be tensioned to apply a vertical compressive load of 10kN, calculated as representative of a two storey masonry infilled timber frame building typically to Tewkesbury.

3. Flood basin enclosing the specimen and providing capability of simultaneous internal and external flooding representative of a flood inundated building.
4. Spray nozzles capable of producing a range of water flow rates and incorporating an air supply such that the spray is atomised to best represent rainwater droplets.
5. Drying fans capable of producing an air flow of 2m/s across the surface of the test specimen, conditions requisite for drying in accordance with BS EN ISO 15927-3. As such no air pressure is simulated in the test procedure.

The water was applied to the face of the specimen using a spray nozzle from which a combination of air and water was dispersed, in order to create a droplet array representative of the wetting of the wall by rain droplets carried horizontally by wind. An actuator controlled the nozzle output, such that it could be programmed remotely and multiple cycles could be run continuously during the 24-hour period, without intervention by a technician.

In total 4 nozzles were used to provide coverage across the whole specimen, with each nozzle producing a spray cone of 60 degrees and situated 50 cm away from the face of the specimen. At maximum capacity each nozzle produces a flow of 1.8 L/minute, with the water atomised evenly across the area projected by the cone onto the surface. Floodwater was introduced to the basin independently by hand direct from the supply, ensuring the water used was free from debris that may have collected within the recirculation system, and also ensured a controlled rise of floodwater depth.

Drying of the wall was carried out using multiple fans generating a cross-flow of air on the surface of the specimens. This was monitored throughout the test using an anemometer to ensure constant conditions. Air temperature was that of the ambient laboratory condition, which fluctuated between 18 and 22 degrees Celsius dependent upon time of day and season. Relative humidity within the laboratory fluctuated within a range of 55 to 65 %. These ambient conditions are recognised as being different from likely external conditions during a wind driven rain event, however manipulation of these were beyond the capacity of the laboratory facility.

Structural Test Procedure

On completion of the weathering test each frame specimen was subjected to a racking test carried out with reference to BS EN 594 (BSI, 2011), with fixing and loading conditions as in Figure 5. The test was carried out in a separate test rig, such that the specimen was no longer subject to any weathering once structural testing had commenced. This is typically used to measure loss of stiffness in timber frames and is therefore a globally recognised parameter of fatigue. The method is limited in its applicability to masonry infilled frames, in that it assumes larger deflections than would occur in a masonry infilled system (100mm), and this defines failure according to the test method. Additionally vertical point loading is specified, which does not account for the unifrom spead loads that a masonry infill will induce onto a timber frame.

In amendment of the standard therefore a vertical compression load of 10kN was applied uniformly across the top rail of the specimen using a hydraulic jack and spreader beam. Meanwile a cyclic in-plane lateral racking load was applied at the top right corner of the frame. In accordance with the standard, racking loads were stabilised for a time period of 300 +/- 60 seconds, alternated with unloaded periods of the same duration. Increments of 1kN load were applied up to 10kN, at which point 2kN increments were used until failure was attained.

Displacement was measured using LVDT sensors, with the frame constrained by the spreader beam at the top, directly underneath which was sandwiched a strip of engineering cork to ensure good friction contact between the steel and the timber. The frame was fixed along its base to the steel base plate by portland cement mortar and was prevented from sliding using a steel restraint at its lower left corner. Timber baton restraints were also fixed to the loading frame, and used to restrict out of plane movement.

The lateral load was applied at the upper right corner of the frame (Figure 5, left), such that the end face of the top horizontal frame member received a point load and the frame was pushed into bending dependent upon the moment capacity of the mortise and tenon joints. Once any joints had mobilised, further loading resulted in sway developing within the timber frame, in addition to bending. Failure was assumed to have occurred when substantial cracking occurred in the masonry and increased loading was impossible. This coincided with a flattening of the backbone load displacement capacity curve for the frame, and so this condition also defined the ultimate load for the frame

The load and displacement data obtained from the test was used to determine the stiffness and strength of the composite timber frame and masonry infill system. The sway nature of the displacement of the frame subjects the infill to diagonal compression loading, and as such the shear stiffness of the masonry is contributory to the stiffness exhibited by the whole system. To quantify this contribution, the effect of exposure to the weathering simulations on the masonry was independently assessed through the testing of masonry wallettes representative of the infill panel, as reported in Stephenson et al., (2016).

Test Specimens

The masonry infilled timber frame system replicated in the laboratory does not incorporate a bracing element (Figure 6). The type of construction used here is often seen in historic frames where a post and beam system is used to transfer loads across multiple bays in a single façade, and where bracing members are provided only at the corners. This particular design is especially vulnerable to racking effects from wind loading, such as would be present during a wind driven rain event due to the lack of bracing. Cyclic loading of the system will instigate sway and eventually permanent deflection as fatigue is instigated by the loading cycles. This therefore represents the worst case building typology with regards structural vulnerability of the system to this particular climatic condition.

The final design of the specimens was derived from structures observed on site in Tewkesbury (Figure 7). Joints between the cross-rails and uprights are constructed with a single oak peg mortice and tenon joint. The design of the frame reflects early English timber frame design, where large principal posts and beams were spaced at greater distances to provide doorways or window apertures (Brunskill, 2006). This also generates a more vulnerable structure, as the beam and post members will be under higher stress conditions due to increased spans and applied loads.

Each timber frame specimen was constructed using reclaimed oak originally used in construction approximately 200 years ago. Pieces were selected for use in the frames based upon their condition; being as much as possible free from knots, sapwood, twist and fungal or insect damage. Grading of the material in accordance with BS 5756, the code for visual hardwood grading (BSI, 2011) determined that the majority of the oak was grade TH1, the higher of the two possible grade outcomes.

The masonry infill was constructed from reclaimed bricks aged to around 1820 and selected due to their high absorption characteristics and their shallow dimension, allowing more courses in the test specimens. The bricks were laid with a non-hydraulic lime mortar in the ratio 1:2.25, as would typically be used at the time such frames were originally constructed (Davey, 1961). The masonry was laid in stretcher bond with a 10mm bed and cured inside the timber frames for a period of one year prior to testing, to allow for optimum strengthening of the mortar bonds. Render was not applied to the specimens to encourage wetting of the masonry and allow thorough investigation of the masonry damage after testing. This also reflects observations of buildings from site where render is missing.

Impact of Weathering on Racking Capacity

Empirical Data

Material Properties

Characteristic physical and mechanical properties of the timber and masonry infil were obtained from samples tested in accordance with the relevant British Standard (Table 1). The elastic modulus of the timber batons is comparable with a hardwood classification of D18, the lowest grade recognised by the UK Timber Classification

Board (TRADA, 2011). The masonry modulus of elasticity is lower than other published empirical historic masonry modulus values, however the weak non-hydraulic lime mortar is likely to have contributed significantly to this. Figures quoted in Table 1 also highlight that historic masonry properties can be highly variable across the world, with the age of the structure also a factor.

Table 1 Nominal and published values of strength and stiffness for timber and masonry

Moisture Uptake

Continuous weighing of the specimens during weathering demonstrated that the rate of uptake was initially very high, but reduced significantly after approximately the first 10 cycles, as exemplified in Figure 8 for Frame 2. The average total moisture content of the three frames ranged between 5% and 6% throughout the weathering test, largely accumulated in the initial uptake period. This is a relatively low moisture content, when compared to the 17% porosity of the bricks for example, highlighting the significance of even low level moisture accumulation on structural integrity.

Crack and Detachment Propagation Mapping

Visual assessment of the decay of the structure carried out on completion of the weathering test used tape measurement of masonry bond loss and infill panel-frame detachment, as shown in Table 2. The percentage of bond loss is a measure of the proportion of the total head and bed joint lengths within each infill panel section. The percentage of detachment is a measure of the proportion of the total possible length based upon the perimeter of the infill panel. In most cases these features were observed on both faces of the frame, especially in the case of bond loss between the frame and infill.

Table 2 Crack and detachment propagation under increasing weathering

Weathering caused considerable detachment to occur between the infill panel and frame for all three scenarios (Frames 2, 3, 4), with the final scenario causing 100% of the panel to detach in both the upper and lower portion. Bond loss also occurs in all scenarios and in both upper and lower panels, however the percentage is much lower than the percentage of detachment. Much greater vulnerability is identified therefore at the interface between the infill and frame, as oppose to in the masonry itself. The extent of detachment or bond loss is not proportional to the number of weathering cycles, possibly as a result of a number of parameters, such as the variation in the reclaimed materials used in the specimens, and the non-linearity in the response of those materials to increased water exposure when working as a composite system.

Racking Tests

The racking test load displacement cycles are presented in Figure 9, and the corresponding load-displacement envelopes are shown in Figure 10. Three phases of behaviour can be identified from the test data: (1) an initial phase of bedding in, with low values of stiffness increasing with displacement, most pronounced in Frames 3 and 4 where significant gapping was induced between the frame and infill as a result of the weathering; (2) a second phase where the stiffness of the composite system is exhibited; (3) a post-yield phase where the stiffness of the system reduces drammatically, following failure of one or more elements.

The elastic stiffness is defined from the first segment of each of the envelopes, and the yield point by the sudden shallowing of the load-displacement curve, indicated by the crosses on the envelopes in Figure 10, at which point the yield strength is recorded and remaining curve defines the yield stiffness. Table 3 sets out these key parameters for the phases of behaviour of the frames. Both the elastic and yield stiffness's are calculated in accordance with the below racking stiffness equation, given in BS EN 594 (BSI, 2011), and which calculates stiffness at the 10th (F_1) and 40th (F_4) percentile of the maximum load, F_{max}. Here F_{max} is defined as the limit of proportionality in the initial elastic region, and the maximum load applied to each frame in the case of yield stiffness.

$$R = \frac{(F_4 - F_1)}{(d_4 - d_1)} \qquad (2)$$

Table 3 Stiffness and strength characteristics of HMITF specimens

Losses in elastic stiffness in the range 79-98% are measured across Frames 2, 3 and 4, demonstrating that weathering over the lifetime of a historic timber frame building causes a substantial loss in structural integrity, even under pre-yield (service) conditions. Less significant losses in yield strength were observed, up to 36% in Frame 4, with Frame 3 recording a yield strength comparable to the unweathered Frame 1. This could be attributed to the weathering impact largely affecting the interfaces between components in the HMITF's, such as masonry bonds, which contribute to overall loss of stiffness, and having less significant impact on the overall strength of the system.

The bedding-in phase which develops at the beginning of structural loading and only after considerable weathering, demonstrates that the potential for damage induced by rocking and sway in these frames in-situ is considerable, and directly attributable to weathering from the environment as oppose to structural fatigue induced by any loading time-history. Further loss of stiffness post-yield is accounted for by the higher loading of the frame during the post-yield phase, and the observation that the joints are being loaded beyond their yield point into permanently deformed states.

Structural cracking was measured throughout the duration of the loading cycles for each frame, with the final observed patterns shown in Figure 11 for Frame 1, whilst Figures 12, 13 and 14 show structural cracking compared to weathering induced cracks for Frames 2, 3, and 4 respectively. The shape of the structural crack pattern observed after the racking tests was similar for all the frames; staggered cracks developed within the mortar joints and travelled diagonally from bottom left to top right of the panels. This is in accordance with a compression load applied at the top left and bottom right (Figure 16), which ultimately leads to tension cracks along the instigated diagonal axis of the panel. For each of the progressively more weathered specimens, cracking extended through the masonry at a lower load level.

Structural cracking was observed in Frame 1 in the load range 18-20kN. This reduced to the range 15-18kN for Frame 2, 8-16kN for Frame 3 and 2-6kN for Frame 4. This indicates a progressive reduction in shear capacity in the masonry, as was also observed in the independent testing of masonry panels exposed to the same weathering regime (Stephenson, Aktas and D'Ayala, 2016). Cracking in Frame 4 as a result of the weathering was so extensive that independent structural cracks did not develop in the masonry as a result of lateral loading. Rather some of the weathering cracks acted as mechanisms for displacement, such as for example the large crack in the top left corner of the lower panel, which opened up on loading of the frame (as shown by the yellow arrows in Figure 14).

Observations made during the testing of Frames 1 and 2 suggest that loss of elastic behavior was as a direct result of bond failure of both head joints and bed joints in the masonry, and that the stiffness exhibited after this point was contributed mainly by the timber elements, and the frictional rotation of the mortise and tenon joints. The joints were observed to rotate and localised crushing of fibres around the tenon was recorded, however rupture of the oak peg holding the joint together did not occur in any of the load tests, only minor permanent bending was observed on their removal after the test (Figure 15). In the following section the empirical evidence for vulnerability is expanded on with an assessment of the timber frames load-deflection behavior.

Assessment of Infilled Timber Frame Behaviour

Analysis of masonry infilled concrete or steel frames often applies the diagonal strut approach (Crisafulli & Carr, 2007), (Nassirpour & D'Ayala, 2017). The assessment developed and presented by the authors here also incorporates this approach, within the following method. The frame is first considered in elastic bending, and the masonry infill is assessed as a shear panel. Shear deformation can cause either horizontal sliding of masonry courses (pure shear), or a combined tension failure in the head joints with shear failure in the bed joints, which instigates the diagonal crack in the masonry. As the system reaches yield condition and deflection increases, failure is either as a consequence of joint rupture as rotation increases, or by diagonal compression in the masonry panel.

At this stage the frame is considered to act as a pin-jointed system, such that the masonry infill acts as a diagonal strut within the frame, loaded in compression. The progression of this failure mechanism is described in Figure 16. In the case of the specimens here the presence of multiple diagonal cracks off the main axis of the panel supports the use of the diagonal strut model for assessing post crack behaviour. Timber connection rupture was not observed in the specimens, and the constant stiffness observed on re-loading of the frame after multiple cycles suggests deformation was as a result of reduced stiffness in the masonry infill alone.

Holmes (1961) was the first to suggest a method for approximating strut width from panel dimensions. Later work by Stafford-Smith (1966) introduced the use of a dimensionless parameter (λ_h), representing the relative stiffness of frame and infill, to calculate the strut width. Mainstone (1971) set out the method for determining strut geometry used here, theorising that following the formation of cracks in the panel two or more struts are assumed to develop in the region bounding the cracks with the width of the effective strut calculated in accordance with Equation 3 below, where d_{inf} and H_{inf} represent the diagonal length and height of the infill panel respectively.

$$w = 0.16\, d_{inf}\left(\lambda_h H_{inf}\right)^{-0.3} \tag{3}$$

Applying this principle, the struts identified in the specimens in this experimental programme are shown in Figure 17. These are used to calculate the dimensions of an effective masonry strut, and in conjunction with reduced masonry modulus are used to calculate the expected deformation in the frame in the post-yield phase. Full detail of this assessment is provided in Appendix A.

Elastic and Pre-Yield Behaviour in Unweathered Frame 1

In the unweathered case the frame is first assessed under pre-yield conditions at 10kN lateral load, so that the stiffness of the system when structurally robust is quantified and the application of the diagonal strut method at yielding is placed within this context. The timber frame is assumed to act as a portal frame with moment transferring joints, and the slope deflection method applied. Meanwhile the masonry panel is considered in combined shear and bending to determine lateral deflection. These two values define the upper and lower bound of expected deflection in the composite system. Full assessment is given in Appendix A, which yields a frame deflection of 4 mm, and shear deflection in the masonry of 0.86 mm. Frame 1 (unweathered) displayed a deflection of 0.8 mm at 10 kN of lateral load, showing the masonry infill dominates behaviour in the elastic range.

Yielding of Frame 1 occurred at 14 kN, at which point the diagonal strut model is assumed to be applicable, with the effective strut width taken as the diagonal length of the panel, according to Mainstone (1971). Applying the principle of virtual work to a pin-jointed truss braced with the masonry strut, the predicted overall deflection is calculated as 1.2 mm (Appendix A), whilst the observed deflection in Frame 1 at yield load was 2mm. Assuming the masonry panel is still acting in combined shear and bending, the masonry deflection is calculated as 1.7mm at 14kN of load. The observed behavior in the unweathered frame suggests that at yielding the masonry stiffness is again dominating behavior of the overall system.

It is also true that a full pin joint likely did not develop in the frame at 14kN, as the geometry of the mortise and tenon joint and the presence of the dowel, generating frictional restraint, prevents free rotation, thus limiting the extent of lateral deflection compared with the theoretical hinge. Detailed assessment of the rigidity of the mortise and tenon joint is beyond the scope of this work, however is clearly an important issue for consideration in future studies (Quinn, D'Ayala, & Descamps, 2016).

Post-Crack Behaviour in Frames 1, 2, 3 and 4

Applying the struts as shown in Figure 17 the theoretical deflection in Frames 1 to 4 is calculated at the lateral load corresponding to maximum cracking, highlighted in the envelopes in Figure 10. The table below compares the deflection observed in the post-yield portion of the envelope up to final cracking load, with the theoretical deflection according to the diagonal struts extracted from the crack patterns, computing also the percentage difference between empirical and analytical values.

Table 4 Comparison of observed and theoretical deflection

The theoretical model computes a deflection that is less than the observed in all cases, by between 2 and 17%. The difference values are comparable with Stafford-Smith's difference of 15%, observed between experimental test and theoretical work when the non-dimensional parameter λ was first proposed (1962). The increase in difference values may be attributable to the variance in timber elastic modulus, which would be larger than in concrete. Additionally, the elastic modulus of the masonry panels may not correspond to the value of E obtained from separate wallet testing carried out for material characterization (Stephenson, Aktas & D'Ayala, 2016)

The comparable results between empirical and theoretical work, demonstrates that the mode of failure in the weathered system is that of the compression strut. The tests and associated calculations demonstrate that the weathering leads to a measureable reduction in racking stiffness as a consequence of loss of bond in the masonry due to this weathering leading to reduced compression strut area. This sets out a quantifiable link between exposure to wind driven rain and flood, and loss of structural integrity in this type of historic construction system.

Vulnerability to Wind Loading

The findings above have further consequences for the resistance of the system to wind loading. The cyclic lateral loading of the timber frames is comparable to the conditions a timber frame building would be subject to during a windstorm. The behaviour of the infilled timber frame is therefore placed in the context of the hazard by converting the loads sustained by the frames into comparable wind loading conditions. Typical UK average (Met Office, 2016) and 0.02 exceedance (50 year return period) wind speeds (BSI, 2005) are shown in Table 5, along with values specific to the case study location of Tewkesbury.

Table 5 Wind speeds for UK and Tewkesbury average and 50 year return periods

In Table 6 the loads applied at the yielding of the timber frames are converted into wind speeds, and compared with the load that corresponds with the 20 m/s wind speed applicable to a 1 in 50 year event in Tewkesbury. The loads are converted back to wind speeds by applying the procedure set out in BS EN 1994-1-4 in reverse, to determine first wind pressure and then wind speed. Terrain, turbulence and other relevant factors are all assumed to be 1, meanwhile the area on which the wind pressure is assumed to act corresponds to the 1.5 m^2 area of the test panels.

The yield loads are higher than the average or storm conditions identified by codified data or national databases. However, they are of comparable size to typical UK gust winds even in low-level zones. For example, the record gust speed for the region in which Tewkesbury is located (Midlands) is 114 mph (50 m/s) for sites below 500m AMSL (Met Office, 2016).

Table 6 Conversion of empirical loadings conditions into generic wind speeds

When considering the 0.02 exceedance value as an equivalent lateral load of 3.75 kN, the increase in displacement displayed by the frames as they are exposed to more severe weathering in notable. In the unweathered sample this load led to a displacement of 0.15 mm, whilst after 100 cycles 1.4 mm of displacement was recorded, and after 200 cycles, 3.25 mm. In the case of Frame 4 where the wetting and drying was extended with longer drying periods, this displacement had increased to 9mm.

This trend is shown in Figure 18 where the displacement is correlated with the total test duration in hours, which was used in order to derive a single measure of hazard severity that could be applied to the different weathering simulations. A second order polynomial relationship is fitted to the data, and highlights that there is an increasing rate of loss observed as the hazard increases, described by the upwards curve of the trend line. Upwards trends in hazard severity as climate change further unfolds indicates that in the future these construction systems are likely to demonstrate ever more increasing levels of loss and damage.

The extent of deflection in Frame 4, coupled with the increased rate of loss over time presents this structural system as highly vulnerable to exposure of this kind. A deflection of 9mm could lead to considerable damage to internal finishes, or instigate further structural damage such as at connections to roof elements. This finding highlights the importance of quantifying these relationships, so that the level of risk associated with the interaction of the structure and the hazards can be identified and its significance presented to both the conservation and engineering communities.

Conclusions

- Structural tests have demonstrated that a cause and effect relationship exists between exposure to wind driven rain and flood and loss of structural integrity in historic masonry infilled timber frames.
- Racking stiffness assessment demonstrates that the weathering alters and reduces the construction system integrity such that the failure mechanism of the system changes from a shear failure to a diagonal compression failure.
- Good correlation is found between weathering crack patterns and loss of stiffness due to diagonal strut geometry change, highlighting that weathering assessment can be used to predict loss of structural integrity in masonry infilled timber frames.
- Assessment under wind loading demonstrates that even under moderate wind conditons loss of stiffness due to weathering leads to substantial deflections in the system, such as would cause secondary damage to buildings and finishes.

- An increasing rate of integrity loss as weathering severity increases is deomnstrated for this construction system, highlighting significant vulnerability of this historic building typology to this climate hazard.

This programme of testing represents one of the first attempts to generate empirical measures of the fragility of traditional brick masonry infilled timber framed structural systems to exposure to flood and wind driven rain hazards. The derivation and execution of the test procedure is in itself novel, meanwhile the test results have highlighted that a significant amount of risk is posed to these structures by such hazards. The findings are not yet conclusive in every regard and are only applicable to the specific materials, masonry and frame system used. However, the data has provided initial quantification of the extent of material and structural degradation caused to this specific structural system by cyclic exposure to wetting and drying, and simulated flood conditions. Furthermore a clear relationship between the loss of yield stiffness and exposure duration has been derived, both from empirical and theoretical methods, which sets out an envelope of fragility in which the system can now be considered.

The findings of the investigation highlight that the structural risks are both real and measureable, although they are derived from a deterministic methodology. It is hoped that this technical information will aid the heritage community in prioritising and managing further mitigation activities, now that the scope and nature of the problem is described quantitatively and in more detail. In addition to assessment of other structural systems, a key next step for the work is the derivation of a probabilistic methodology which achieves the same physical assessment goals. This will ensure that the findings of the structural and weathering analysis procedures are incorporated into future risk assessment protocols surrounding the impacts of wind and precipitation on historic timber frame structures, the pursuance of which will promote and progress risk reduction goals for the heritage community.

Appendix A

Elastic Phase – Slope Deflection Assessment

First the elastic case is considered using the slope deflection method (Figure 19), applying the general equation:

$$M_{ij} = \frac{2EI}{L}\left(2\theta_i + \theta_j\right) + \frac{WL^2}{12}$$

(4)

Applying horizontal and vertical equilibrium gives:

$$M_{AC} + M_{AB} = 0$$

$$M_{BA} + M_{BD} = 0$$

$$M_{CA} + M_{CE} + M_{CD} = 0$$

$$M_{DB} + M_{DC} + M_{DF} = 0p$$

$$M_{AC} + M_{CA} + M_{BD} + M_{DB} = 7.5$$

$$M_{CE} + M_{EC} + M_{DF} + M_{FD} = 7.5$$

(5)

Substituting (4) into (5) and applying Gaussian Elimination produces the following set of equations written in matrix format:

$$\mathbf{EI}\begin{bmatrix} \frac{28}{3} & 2 & \frac{8}{3} & 0 & \frac{96}{9} & 0 \\ 2 & \frac{28}{3} & 0 & \frac{8}{3} & \frac{96}{9} & 0 \\ \frac{8}{3} & 0 & \frac{44}{3} & 2 & \frac{96}{9} & \frac{96}{9} \\ 0 & \frac{8}{3} & 2 & \frac{44}{3} & \frac{96}{9} & \frac{96}{9} \\ 8 & 8 & 8 & 8 & \frac{384}{9} & 0 \\ 0 & 0 & 8 & 8 & 0 & \frac{384}{9} \end{bmatrix}\begin{bmatrix} \theta_A \\ \theta_B \\ \theta_C \\ \theta_D \\ \delta_1 \\ \delta_2 \end{bmatrix} = \begin{bmatrix} \frac{-10}{12} \\ \frac{-10}{12} \\ 0 \\ 0 \\ 7.5 \\ 7.5 \end{bmatrix} \tag{6}$$

Using values of E from the timber baton tests (Table 1), I is calculated from a member cross section of 120 x 120 mm, giving I = 17.28 x 10^6 mm. Solving for values of θ and δ gives:

$\theta_A = -0.00356\, rad$ $\quad\quad$ $\theta_B = -0.00356\, rad$

$\theta_C = -0.00389\, rad$ $\quad\quad$ $\theta_D = -0.00389\, rad$

$\delta_1 = 4.2\, mm$ $\quad\quad$ $\delta_2 = 2.8\, mm$

Rotational values of θ are converted to lateral movement through multiplication by the height of the sway mechanism, which is 750 mm in the case of these frames. This gives a total deflection in the frame of 4 mm.

Calculating the deflection of the masonry infill in combined shear and bending, using the formula given by Hendry et al. (1997) for a panel constrained only at the base:

$$\delta = \frac{Wh^3}{3EI} + \frac{\lambda Wh}{A\,G} \tag{7}$$

where the height (h), shear area (A) and second moment of area (I) are calculated from the panel dimensions of 575 x 820 x 120 mm. The shear modulus (G) equals 40%

of *E* (BSI, 2012), giving G = 164 N/mm^2 and λ = 1.2. This gives the predicted shear deflection of the masonry as 0.86 mm, indicating that the masonry stiffness dominates the overall racking stiffness in the pre-yield state.

Post Crack Phase - Diagonal Strut Assessment

The diagonal strut model (Figure 20) is applied and the pin-jointed truss assessed using virtual work, for five cases: Frame 1 at pre-yield and post-crack, and Frames 2, 3 and 4 at post-crack.

Deflections are computed below for estimated strut geometries as shown in Figure 17. The initial masonry stiffness is calculated from compression testing of the masonry units and mortar, according to Eurocode 6 (BSI, 2012). This gives a compressive strength for the masonry of 4.1 MPa, an acceptable value for historic masonry.

Converting to the elastic modulus requires the application of a constant, K_E. Reporting on appropriate values of K_E range from 1000 (EC6), through to observed values as low as 250 (Narayanan & Sirajuddin, 2013). The observed value of stiffness calculated from tests by these authors for masonry panels tested in isolation was 163 MPa, corresponding to a K_E value of 40. For analysis however, a more conservative K_E value of 100 is used. For each weathered frame an estimated reduced value of *E* is used, reflecting the loss of stiffness exhibited by the masonry when tested separately in combined compression and lateral loading (Stephenson, Aktas & D'Ayala, 2016). The masonry modulus values used are:

Frame 1: E = 410 N/mm^2 Frame 2: E = 382 N/mm2, Frame 3: E = 382 N/mm2 Frame 4: E = 210 N/mm2

The strut width is defined by Stafford-Smith and Carter (1969) and Mainstone (1971) using the following equations, where λ_h is a dimensionless parameter accounting for the relative stiffness of the infill panel and frame, and w is the width of the strut:

$$\lambda_h = \sqrt[4]{\frac{E_{inf} t \sin 2\theta}{4 E_c I_c H_{inf}}} \tag{8}$$

$$w = 0.16\, d_{inf} \left(\lambda_h H_{inf}\right)^{-0.3} \tag{9}$$

where:

E_{inf} = Panel Elastic Modulus t = Panel Thickness

H_{inf} = Panel Height θ = Angle Panel Diagonal to Horizontal

E_c = Frame Elastic Modulus I_c = Frame Moment of Inertia

d_{inf} = Diagonal Length of Strut

For each identified strut (shown in Figure 17) the diagonal length is measured (d_{inf}) and the associated strut width calculated from Equation 9.

Table 7 Deflection of Frame 1 at yield load (14kN)

Table 8 Deflection of Frame 1 at post-cracking (21kN)

Table 9 Deflection of Frame 2 at post-cracking (18kN)

Table 10 Deflection of Frame 3 at post-cracking (16kN)

Table 11 Deflection of Frame 4 at post-cracking (6kN)

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Table 1 Nominal and published values of strength and stiffness for timber and masonry

Property	**Value (MPa)**	**Standard/Reference**
Timber Flexural Strength	64	BS 373
Timber Elastic Modulus in Bending	7486	BS 373
Masonry Shear Strength	0.105	EC 6
Masonry Elastic Modulus	163	BS EN 594
Masonry Shear Modulus	65	EC 6
Elastic Modulus_Istanbul (19^{th} C)	2500	Aras & Altay (2015)
Elastic Modulus_Khatmandu (18^{th} C)	274	Parajuli (2012)

Table 2 Crack and detachment propagation under increasing weathering

Frame Number	**Infill Detachment** Damage (%)		**Masonry Bond Loss** Damage (%)	
	Upper Panel	Lower Panel	Upper Panel	Lower Panel
Frame 1	0	0	0	0
Frame 2	75	69	10	3
Frame 3	49	60	7	6
Frame 4	100	100	2	7

Table 3 Stiffness and strength characteristics of HMITF specimens

Frame Number	Elastic Stiffness (N/mm)	Loss of Elastic Stiffness (%)	Yield Strength (kN)	Loss of Yield Strength (%)	Yield Stiffness (N/mm)	Loss of Yield Stiffness (%)
1	12000	-	14.0	-	330	-
2	2536	79	12.0	14	440	-
3	1071	91	14.0	-	280	15
4	200	98	9.0	36	0	100

Table 4 Comparison of observed and theoretical deflection

Frame No.	Final Crack Load (kN)	Observed Deflection (mm)	Theoretical Deflection (mm)	Percentage Difference (%)
1	20	16	13.3	-17
2	18	11.5	11.3	-1.7
3	16	12	10.0	-17
4	6	8	7.7	-3.8

Table 5 Wind speeds for UK and Tewkesbury average and 50 year return periods

Wind Condition	Wind Speed (m/s)
UK_Mean	6.5
UK_0.02	25.5
Tewkesbury_Mean	3.5
Tewkesbury_0.02	20

Table 6 Conversion of empirical loadings conditions into generic wind speeds

Frame Number	Load (kN)	Wind Pressure (kN/m2)	Wind Speed (m/s)
1	15	10	40
2	12.5	8.3	36.5
3	14	9.3	38.6
4	9	6	31.0
Tewkesbury 0.02	3.75	2.5	20.0

Table 7 Deflection of Frame 1 at yield load (14kN)

Member	**E (N/mm2)**	**Length (mm)**	**Area (mm2)**	**Force (kN)**	**Deflection, δ (mm)**	**Unit Force F_u**	**Final Deflection, $F_u.\delta$ (mm)**
AB	7486	1000	14400	0	0.0000	1	0.00
AC	7486	750	14400	-5	-0.0348	0	0.00
CB	410	1000	120000	-17.5	-0.3557	-1.25	0.44
BD	7486	750	14400	5.5	0.0383	0.75	0.03
CD	7486	1000	14400	14	0.1299	1	0.13
CE	7486	750	14400	-15.5	-0.1078	-0.75	0.08
ED	410	1000	120000	-17.5	-0.3557	-1.25	0.44
DF	7486	750	14400	15.5	0.1078	0.75	0.08
					Total Deflection		1.21

Table 8 Deflection of Frame 1 at post-cracking (21kN)

Member	**E (N/mm2)**	**Length (mm)**	**Area (mm2)**	**Force (kN)**	**Deflection, δ (mm)**	**Unit Force Fu**	**Final Deflection, Fu.δ (mm)**
AB	7486	1000	14400	0	0.0000	1	0.00
AC	7486	750	14400	-5	-0.0348	0	0.00
CB	410	1000	12720	-26	-4.9854	-1.25	6.23
BD	7486	750	14400	10.6	0.0737	0.75	0.06
CD	7486	1000	14400	20.8	0.1930	1	0.19
CE	7486	750	14400	-20.6	-0.1433	-0.75	0.11
ED	410	1000	11160	-26	-5.6823	-1.25	7.10
DF	7486	750	14400	26.2	0.1823	0.75	0.14
					Total Deflection		13.83

Table 9 Deflection of Frame 2 at post-cracking (18kN)

Member	**E (N/mm2)**	**Length (mm)**	**Area (mm2)**	**Force (kN)**	**Deflection, δ (mm)**	**Unit Force F_u**	**Final Deflection, $F_u.\delta$ (mm)**
AB	7486	1000	14400	0	0.0000	1	0.0000
AC	7486	750	14400	-5	-0.0348	0	0.0000
CB	382	1000	12600	-22.5	-4.6746	-1.25	5.8433
BD	7486	750	14400	8.5	0.0591	0.75	0.0444
CD	7486	1000	14400	18	0.1670	1	0.1670
CE	7486	750	14400	18.5	0.1287	-0.75	-0.0965
ED	382	1000	14040	-22.5	-4.1952	-1.25	5.2440
DF	7486	750	14400	22	0.1531	0.75	0.1148
					Total Deflection		11.32

Table 10 Deflection of Frame 3 at post-cracking (16kN)

Member	**E (N/mm2)**	**Length (mm)**	**Area (mm2)**	**Force (kN)**	**Deflection, δ (mm)**	**Unit Force F_u**	**Final Deflection, $F_u.\delta$ (mm)**
AB	7486	1000	14400	0	0.0000	1	0.0000
AC	7486	750	14400	-5	-0.0348	0	0.0000
CB	382	1000	13440	-20	-3.8955	-1.25	4.8694
BD	7486	750	14400	7	0.0487	0.75	0.0365
CD	7486	1000	14400	16	0.1484	1	0.1484
CE	7486	750	14400	-17	-0.1183	-0.75	0.0887
ED	382	1000	13440	-20	-3.8955	-1.25	4.8694
DF	7486	750	14400	19	0.1322	0.75	0.0991
					Total Deflection		10.11

Table 11 Deflection of Frame 4 at post-cracking (6kN)

Member	**E (N/mm2)**	**Length (mm)**	**Area (mm2)**	**Force (kN)**	**Deflection, δ (mm)**	**Unit Force F_u**	**Final Deflection, F_u.δ (mm)**
AB	7486	1000	14400	0	0.0000	1	0.00
AC	7486	750	14400	-5	-0.0348	0	0.00
CB	210	1000	11760	-7.5	-3.0369	-1.25	3.80
BD	7486	750	14400	0.5	0.0035	0.75	0.00
CD	7486	1000	14400	6	0.0557	1	0.06
CE	7486	750	14400	-9.5	-0.0661	-0.75	0.05
ED	210	1000	11760	-7.5	-3.0369	-1.25	3.80
DF	7486	750	14400	4	0.0278	0.75	0.02
					Total Deflection		7.72